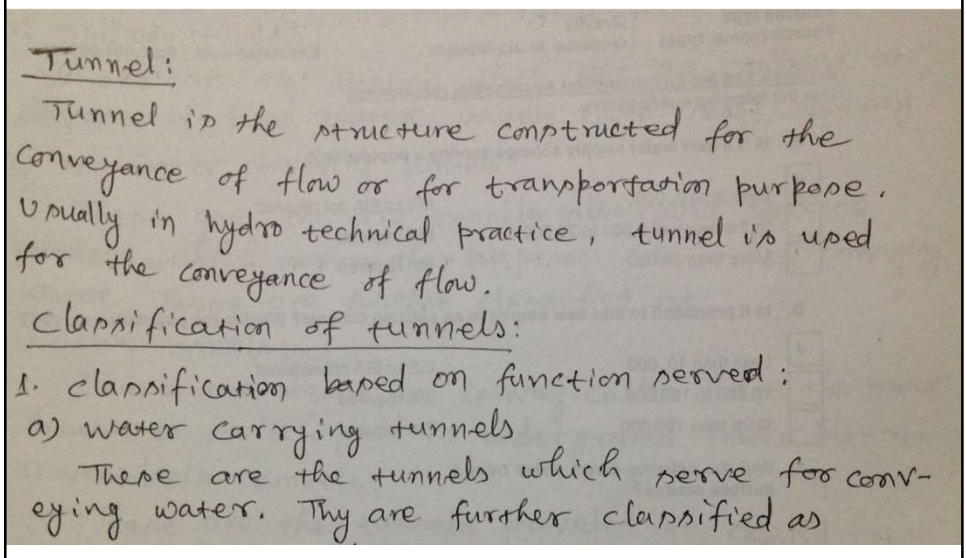
Water Conveyance Structures



Tunnel:

Tunnel is the structure constructed for the conveyance of flow or for transportation purpose. Usually in hydro technical practice, tunnel is used for the conveyance of flow.

Classification of tunnels:

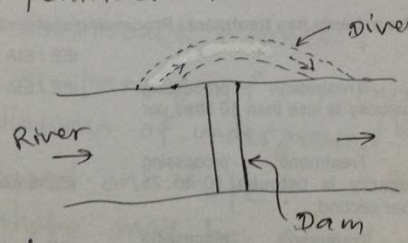
1. classification based on function served:

a) water carrying tunnels

These are the tunnels which serve for conveying water. Thy are further classified as

i) Diversion tunnels:

These are the tunnels constructed to divert the river flow away from the construction area. On completion of the dam these are converted into spillway tunnels / penstock tunnels.



ii) Headrace tunnels/Power tunnels / Pressure tunnels:

These are the tunnels which convey water from dam / barrage to the power plant.

iii) Tailrace tunnels:

These are the tunnels which carry away the water from power plant. These are usually free flow tunnels. When the turbine setting is below tail water level, these are pressure tunnels.

iv) Spillway tunnels:

These are the tunnels which act as spillway to dispose of flood water, usually called shaft spillway.

b) Non-water conveying tunnels:

These are the dry tunnels also called service tunnels which serve the purposes other than conveying water. These are further classified as

i) Cable tunnels:

These are meant to convey cables from the power house to switch yard in underground power plants.

ii) Ventilation tunnels:

These are the tunnels provided to convey fresh air to the underground tunnels.

iii) Adit tunnels:

These are provided to facilitate starting tunnelling at various heads along main tunnel.

iv) Access tunnels / Approach tunnels:

These serve as passage from surface to underground installations of underground power plant.

2. Classification based on shape:

The tunnels shape may be circular, rectangular or any combination of these forms such as horse shoe shape, D-shapped, egg shaped, etc. Circular tunnels are structurally more suitable and more stable. Horse-shoe shape tunnels are very convenient from the construction point of view.

3. Classification based on protective surface:

i) Lined tunnel:

Lining is a protective layer of concrete, RCC or steel on the inner surface of the tunnel. The power tunnels are usually lined to economise on loss of head, smaller cross-section and hydraulically efficient.

ii) Unlined tunnel:

Tunnels in good sound rock can be left unlined.

4. Classification based on alignment:

i) Tunnel:

The term tunnel implies here the one with a slight bed slope or vertually a horizontal tunnel.

ii) Shaft:

It is a tunnel with alignment vertical. Inclined shaft is one which is steeply inclined to the horizontal.

5. Classification based on characteristics of flow:

i) Low pressure tunnel:

It is a tunnel with low internal water pressure i.e, water head less than 10 m. Diversion tunnels are usually low pressure tunnels.

ii) Medium pressure tunnel:

It is a tunnel with water head of 10 to 100 m.

iii) High pressure tunnel:

High pressure tunnel has high internal pressure corresponding to a head of over 100 m.

iv) Drainage tunnel:

It is a tunnel provided in the abutments of an earthen dam to collect seepage water from rainfall, reservoir for safe disposal.

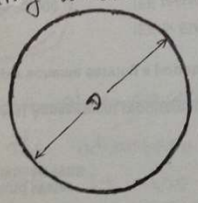
Advantages of tunnels:

- Provides direct and short route for the water passage resulting in saving in head thereby results in saving in project cost.
- Tunneling work can be started simultaneously at many points, facilitating quicker completion.
- Natural landscape of the area is not distrubed
- Tunneling is getting easier and faster with the development of latest tunnel boring techniques.
- The concept of rock mechanics and the latest tool of stress analysis by using finite element method, has established structural stability of tunnels.

Shape of tunnel / Geometric design :

i) Circular section:

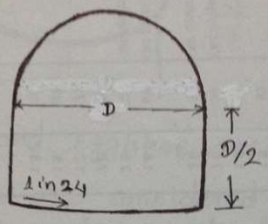
It is the most suitable and stable shape from structural considerations where the tunnel is subjected to high inward or outward radial pressures. For poor quality rock and/or inadequate rock cover around, it is most suitable from the excavation point of view; particularly where cross-sectional area is small.



- Max^m flow occurs, when $\frac{\text{depth}}{\text{diameter}} = 0.93$
- when depth = 0.8 * dia., flow is same as for full circle.

ii) D - shaped section:

It is suitable in good quality, intact sedimentary rocks and massive external igneous hard, compacted, metamorphic rocks where the external pressure due to rock and water are not very large and also where the lining is not designed against internal pressure. Its flatter width of invert gives more working floor space in the tunnel and eliminates the tendency of wet concrete to slump.

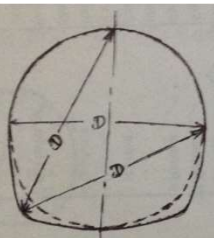


Area = $0.905\ D^2$

Perimeter = $3.58\ D$

iii) Horse-shoe section:

It is a compromise between circular and D-shaped sections. It is structurally strong to withstand external rock and internal water pressures. It is suitable in a moderately good rock. The maximum flow occurs when ratio of depth to diameter is 0.94 and the flow is same as for full section when depth = 0.82 * diameter.

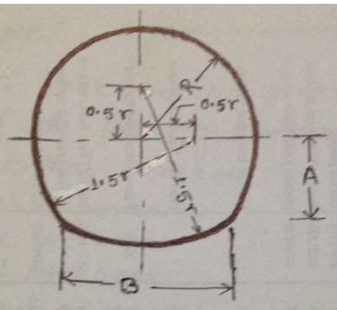


Area = $0.8293\ D^2$

Perimeter = $3.267\ D$

iv) Modified horse-shoe section:

It provides the advantage of a flatter invert for constructional ease. It also affords easy change over to circular section with minimum additional cost in reaches where rock quality is poor or rock cover is inadequate.



Area = $3.254\ r^2$

Perimeter = $6.426\ r$

$A = 0.7808\ r$

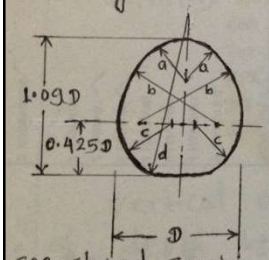
$B = 1.562\ r$

$r = 0.987580\ R$

v) Egg-shaped and Egglipse section:

These sections are considered where the rock is stratified, soft and very closely laminated (as laminated sand, stone, slates, micaceous schists, etc.) and where the external pressures and tensile forces in the crown are likely to be high so as to cause serious rock falls. These are suitable for tunnel carrying sediments.

Egglipse shape has advantage over egg shaped section as it has a smooth curvature and is hydraulically more efficient.

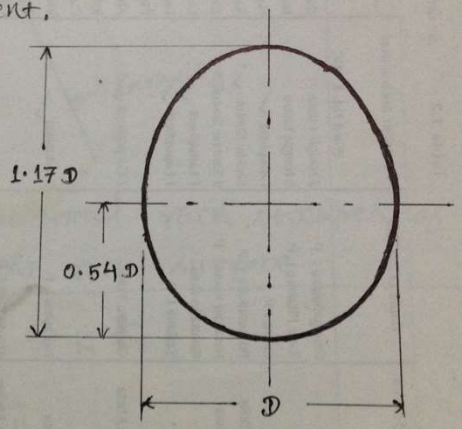


Egg shaped section

a = 0.384D
b = 0.78D
c = 0.45D
d = 1.3D

Area = $0.864\,D^2$

Perimeter = 3.317D



Egglipse section

Loads on tunnel:

- Earth pressure
- Reaction force due to the elastic deformation of the rock
- Over burden pressure
- Hydrostatic pressure
- Earthquake pressure

Tunnel lining:

The power tunnels which connect directly to the penstock are essentially lined with cement concrete or reinforced or steel liner.

Tunnel lining is restored to when

- internal pressure is high (i.e. > 100 m water head).
  → in sound rock formations, ulined upto 200 m water head.

- in strata of low strength and where the rock is anisotropic
- the hydraulic resistance is desired to be low.

Advantages:

- Provides strength and stability and facilitates tunneling in weaker strata.
- Distributes internal loads uniformly in the surrounding rock mass.
- Saves energy by reducing hydraulic resistance.
- Saving in cost with the reduction in cross-sectional area of tunnel.
- Minimises the danger of accidental rock falls in the tunnel cavity
- Seepage across the lining becomes negligible
- Rock trap provision on the upstream of turbines is not necessary.

Types of lining:

1. PCC lining:

Tunnels are lined with PCC when the tunnel walls are to support outside pressure due to the rock, but low pressure inside.

2. RCC lining:

Tunnels are lined with RCC when the internal pressures are high, PCC lining is inadequate and tension cracks are developed in lining.

3. Steel lining:

The steel lining consists of a steel plate of adequate thickness conforming to boiler and pressure vessel steel provided to the inner surface of the tunnel.

The main function of steel lining is to protect the concrete and to stop leakage of water from the tunnel.

It is essentially required where

- The tunnel has to withstand high pressure
- The rock cover is inadequate
- Cavitation of lining is expected due to high velocity of water
- Erosion can occur as in silt flushing tunnel.

4. Shotcrete lining:

Shotcrete lining is provided to protect and support zones of fractured, crushed, disintegrated or spalling rock, and to preserve and prevent further deterioration caused by action of water or atmosphere or aging. Shotcrete lining offers an effective, economical and safe alternative where conventionally placed concrete is not possible.

Structural Analysis:

Because of the applied external rock pressure, hydrostatic pressure, hydrostatic pressure from inside the tunnel and self weight, bending stress as well as compressive stress is developed in the tunnel. Due to that reason, tunnel lining cross-section will be deformed and elastic reaction may be developed.

Calculation of deformation of the tunnel membrane could be done through the known value of the bending moment and the compressive force. With the known value of the deformation, the elastic reaction could be estimated with the following relationship,

$\sigma_{elastic} = K * \delta$

where, δ = deformation in m.

and K = elastic resistance of the rock in KN/m^2, which could be established through in-situ test or from Galerkin's relationship.

Galerkin's relationship,

$$K = \frac{E}{r(1+M)}$$

where, E = modulus of elasticity of rock

r = radius of tunnel

and M = poisson's ratio.

For the approximate calculation value of K for f_h ranging from 1.5 - 20, following emperical relationship can be used.

$$K_o = 500 \alpha f_h \ (kgf/cm^2)$$

$$K = K_o \frac{100}{r_e}$$

where, r_e = radius of tunnel

α = crack intensity coefficient

= 0.8 for highly fractured rock (i.e; $M_q = 5-15$)

= 1 for fractured rock (i.e; $M_q = 1.5-5$)

= 1.2 for less fractured rock (i.e; $M_q < 1.5$)

and f_h = hardness coefficient of rock.

Hydraulic Design:

Losses:

a) Friction losses:

i) Manning's formula

$$h_f = \frac{n^2 v^2 l}{R^{4/3}}$$

where, n = Manning's roughness coefficient

= 0.040 - 0.060 for very rough surface

= 0.025 - 0.035 for surface trimmed

= 0.020 - 0.030 for surface trimmed and

where, n = Manning's roughness coefficient
= 0.040 – 0.060 for very rough surface
= 0.025 – 0.035 for surface trimmed
= 0.020 – 0.030 for surface trimmed and invert concreted.
v = mean velocity of flow
l = length of tunnel
and, R = hydraulic radius or hydraulic mean depth

ii) Darcy weisbach formula

$$h_f = \frac{f l v^2}{2g D}$$

where, f = Darcy's friction factor
l = length of tunnel
v = mean velocity of flow
and D = diameter of tunnel.

b) Trash rack loss:

$$h_t = K_t \frac{v^2}{2g}$$

where, K_t = loss coefficient for trash rack
and v = velocity through contracted opening

$$k_t = 1.45 - 0.45\frac{a_n}{a_t} - \left(\frac{a_n}{a_t}\right)^2$$

where,
a_n - net area through the trash rack bars
a_t - gross area of the opening

c) Entrance loss:

$$h_e = K_e \frac{v^2}{2g}$$

where, K_e = loss coefficient for entrance and assumed to vary from 0.1 for gradual contraction to 0.5 for abrupt contraction

and v = velocity of flow

d) Bend loss:

$$h_b = K_b \frac{v^2}{2g}$$

where, K_b = bend loss coefficient

$$= \frac{2\alpha}{\pi^2 \left(\ln \frac{R_c}{D} + \alpha \right)}$$

α = deflection angle

R_c = radius of curvature

and v = velocity of flow.

e) Grate loss:

$$h_g = K_g \frac{v^2}{2g}$$

where, K_g = gate loss coefficient

= 1.0 for full gate opening

= 1.15 for 3/4 " "

= 5.6 " 1/2 " "

= 24.0 " 1/4 " "

and v = flow velocity

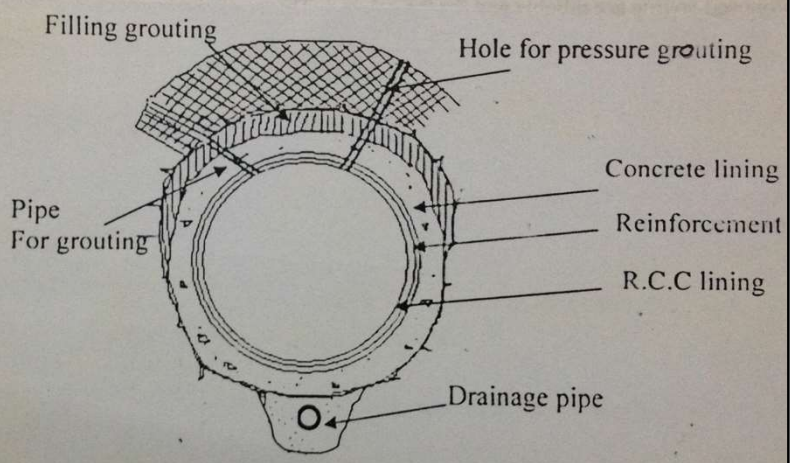
f) Exit loss:

$$h_{exit} = K_{exit} \frac{v^2}{2g}$$

where, K_{exit} = exit loss coefficient ≈ 1.0

and v = flow velocity.

Typical section of pressure tunnel:



Pressure Tunnel

Economic dimensioning of tunnel:

1. Economical diameter of tunnel:

According to 'Handbook of Hydroelectric Engineering' – P.S. Nigam.

$$D^{7.33} = \frac{19.35\, Q^3 n^2 e u * 10^5}{(E + 0.36 L) * O}$$

where,

Q = Discharge
n = rugosity coefficient
e = overall efficiency
u = unit cost of power ($\simeq$ Rs. 7/unit say)
L = unit price of lining ($\simeq$ Rs. 4000/m^3 say)

E = unit price of tunnel excavation

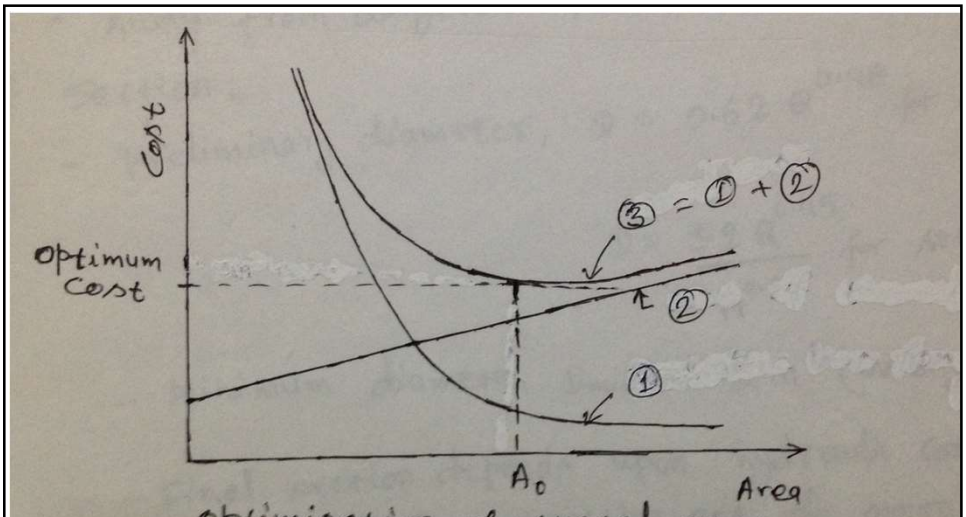
O = Annual operation and maintenance cost
≒ 8% of total cost.

2. Cost benefit analysis:

$\frac{dC_T}{dA} + \frac{dC_B}{dA} = 0$, will give economical area

C_T = cost of tunnel per metre

C_B = Capitalized value of annual head loss



Optimisation of tunnel cross-section

① → Annual power loss
② → Annual cost of tunnel
③ → Total cost

Considering head loss and energy regained, the economic section of the tunnel can be determined based on the optimization of cost of the tunnel and minimizing the energy losses due to friction in tunnel as shown in graph above.

Design considerations:

1. Alignment:
   - As short as economically possible length.
   - Should follow the location of intake on one hand. the surge shafts on the other hand.
   - Availability of convenient adit points.
   - Good and sufficient rock cover.
   - Away from dangerous fault planes.

2. Section:
   - Preliminary diameter, $D = 0.62\,Q^{0.48}$ for concrete lining

     $D = \frac{1.2\,Q^{0.45}}{H^{0.12}}$ for steel lining
   - Minimum diameter, $D_{min} \geq 2$ m (Masonyi ≥ 1.8 m)
   - Final section depends upon hydraulic considerations, convenience of construction and the overall economy of the operation.

3. Slope:
   - Slope is fixed upon the basis of dewatering requirements.
   - Power tunnel with a gentle slope has relatively low internal pressure.

4. Velocity:
- Limiting velocity in unlined tunnels → 2 to 2.5 m/s
- " " " concrete lined " → 4 to 5 m/s
- " " " steel-lined " → 9 m/s
- Greater velocities in diversion tunnels (Mossyrock, USA, planned for max^m velocity of 7.5 m/s)
- Steel lined pressure shafts have higher velocity than power tunnel (normal range of 5 to 8 m/s).

5. Overburden:
- Based on elementary statical equilibrium,

$$\gamma h \leq \gamma_r h_r$$

where, γ and γ_r = unit weights of water and rock respectively

h and h_r = internal water pressure head and desireable height of overburden reppectively.

Taking $\gamma = 10\ KN/m^3$ and $\gamma_r = 25\ KN/m^3$ $[\gamma_r \simeq 24\text{-}32\ KN/m^3]$

$10h \leq 25\ h_r$

i.e. $h \leq 2.5\ h_r$

Taking factor of safety = 2.5

$$\boxed{h \leq h_r}$$ → Thumb rule.

Practical value of factor of safety = 4 to 6 with smaller value for greater depth of overburden.
- Mononyi recommends

$$\boxed{h = 0.4 \text{ to } 0.8\ h_r}$$, depending upon the rock formation.

If $h > 0.4$ to $0.8\ h_r$, lining is required.

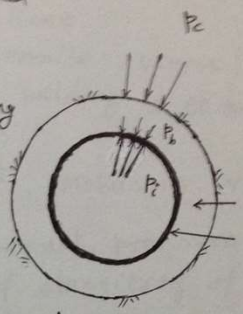
6. Lining:
- Thickness depends upon the load shared by the lining.

- Total internal pressure = pressure transferred to rock + pressure resisted by the lining itself.

P_c = pressure transferred to the rock

$P_b - P_c$ = pressure taken by the concrete lining

$P_i - P_b$ = pressure taken by the steel lining



Stress development in the lining

- when the tunnel is running full, the lining and rock must be able to resist internal pressure of water
- when the tunnel is empty, the lining must be able to resist external load transferred to it.
- Thickness of PCC lining = 20 to 60 cm, poorer the rock, greater will be the thickness.

- Thickness of steel liner (t),

i) Thumb rule

$$t_{min} = \frac{d + 50}{400} \text{ in cm}$$

where, d = internal dia. of tunnel in cm.

$t = t_{min} + 2$ mm (allowance for corrosion)

ii) Estimating actual internal or external pressures:

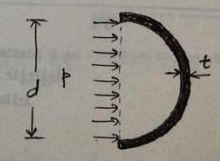
a) Internal pressure:

Internal pressure force = $p.d$

Resisting force = $\sigma_{st} \times 2t \times \eta$

Equating these,

$$t = \frac{p\,d}{2\,\sigma_{st}\,\eta}$$



where, p = the part of the internal pressure to be resisted by liner

d = internal diameter of the tunnel

t = thickness of liner

σ_{st} = permissible tensile stress in steel

η = efficiency of welding ≈ 90 to 95 %

b) External pressure:

$P_{ext} = \frac{8EI(K-1)}{d^3}$, Mobussin relation

where, P_{ext} = design external pressure

E = modulus of elasticity of steel

I = moment of inertia per unit length of the liner $= \frac{t^3}{12}$

t = thickness of liner

d = internal diameter of liner

and K = a factor depending upon the angle of buckling, for an angle of 30°, $K = 8.62$.

Buckling due to external loads can be prevented to a large extent by

- → using dense concrete between the rock mass and liner.
- → grouting deep into rock mass.
- → providing anchorage for the steel liner into concrete backing.

Tunneling Methods:

1. Conventional method
2. NATM (New Austrian Tunneling Method)
3. TBM (Tunnel Boring Machine) method

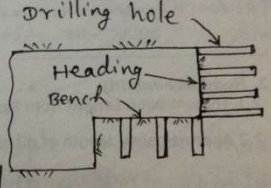
1. Conventional method:

i) Full face method:

In this method, the entire cross-sectional area of the tunnel to be excavated is attacked simultaneously. It is generally adopted for small size tunnels in good rock conditions where major rockfalls are not anticipated.

ii) Top heading and benching method:

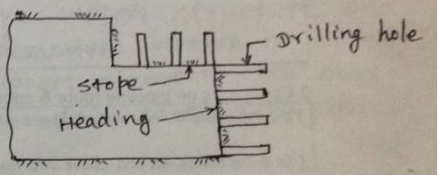
In this method, a top heading is excavated first, either to full length or part length of the tunnel, and is supported simultaneously. The benching is then removed



slowly. This method is used where the tunnel has large cross-sectional area or where the rock is not of good quality.

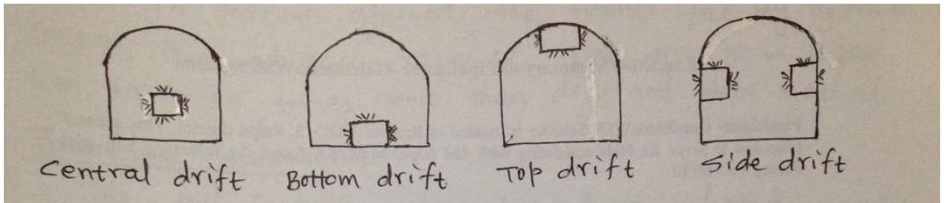
iii) Bottom heading and stoping method:

In this method a bottom heading is made first and the overhead stope is removed later. It is used where the rock is consistent and sound and the tunnel section is very large.



iv) Drift method:

This method is used where in a large tunnel it is economical to drive a small tunnel, called drift or a pilot tunnel, prior to excavating a full face. The drift may be excavated in the centre, bottom, top or side.



Specially the side drift is adapted to large size tunnels through bad rock which require support before mucking.

v) Multiple drift method:

It is a combination of the side drifts and top drift. It is suitable for large openings in crushed rock formations in fault zones. In this method, a side drift is driven first on each side.

2. NATM:

It is based on the principle to take maximum advantage of rock's capacity to support itself by careful measures and deliberate guidance of the force during the readjustment process from primary to the secondary state of stress around the excavations.

The NATM constitutes a method by which the surrounding rock or soil formation of a tunnel are integrated into an overall ring like support structure and as such these formations themselves become a part of the primary support. The support elements are shotcrete, welded wiremesh, rock bolts, steel ribs, forepoling, steel lagging.

3. TBM method:

A full face TBM consists of a wheel cutter head fitted with teeth or rollers to cut or spall the rock. The wheel may consist of spokes or of a solid disc with slots to allow the muck to pass through. The rotating speed of the wheel is 4 to 10 rpm. The wheel is forced against the tunnel face by hydraulic jacks. The advance rates of TBM are 50 m a day i.e, 10 to 15 times more than drill and blast method.

Tunneling sequence:

The full-face tunneling procedure adopted may be carried out in the following sequence:

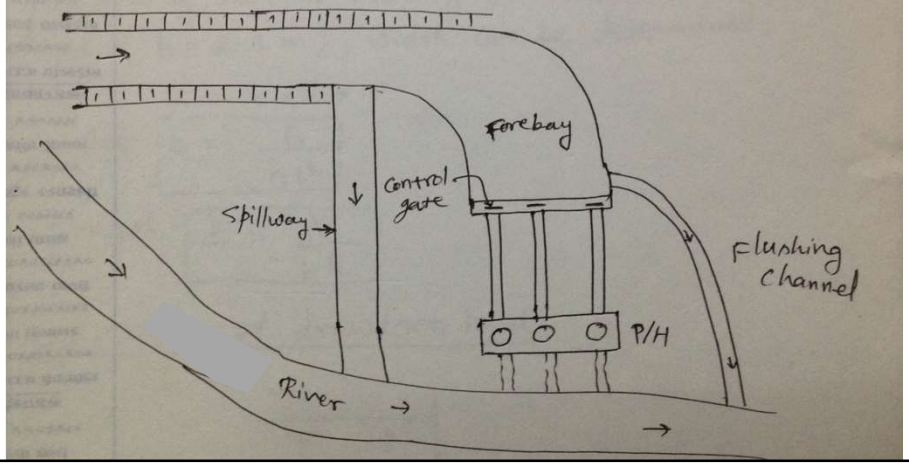
1. Drilling
2. Charging and blasting
3. Ventilation for the exhaust of the fumes cushion
4. Scaling
5. Mucking and Haulage of rock pieces
6. Ribbing (fixing the steel supports)
7. Initial concreting (concreting the space between rock and steel supports.

For the first, a fully mechanised drilling jumbo is required. However, all the first four operations may also be done by the same machine. ventilation may be achieved through fans or air ducts. Sometimes a mixture of compressed air and water blown through the nozzles, brings down the fumes quickly and cleans the air.

After the blasting is over and the fumes are exhausted, any loose pieces are scaled down and the blasted rock is mucked with the help of muckers and mine cars. Then ribbing is done, depending upon the size of the tunnel and the quality of rock material. The gap between the ribs and the rock is then filled with concrete at pressure (shotcrete).

Forebay or Headpond:

It is the pondage at the beginning of a penstock leading water to turbine or the enlarged end of hydel channel in case of a canal as a headrace conveyance.



Main purposes of forbay:

- To distribute discharge evenly among the penstocks / pressure shafts
- To regulate the flow entering into the penstocks.
- To dispose excess water
- To provide daily storage (in some cases) e.g. Sunkoshi

Storage capacity of a forebay:

The flow in the canal is fully accelerated only after sometime of turbine startup. Therefore, enough storage is required in the forebay to generate power during the time taken by the flow for full acceleration. Usually, time depends on site conditions and layout of the system. In

usual considerations duration of 3-4 minutes is considered for the estimation of storage capacity of forebay.

Storage volume, $V = Q * t$

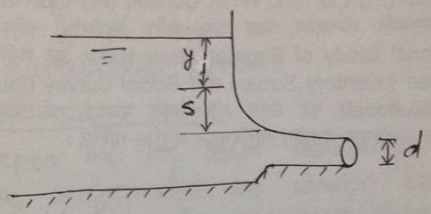
Assuming suitable velocity ($v = 0.1 - 0.2$ m/s) and depth ($h = 2-3$ m), width can be determined.

$$Q = b * h * v$$

$\therefore$ width; $\boxed{b = \frac{Q}{hv}}$

Length, $\boxed{l = \frac{V}{bh}}$

Estimation of drawdown level:



The inlet section of the penstocks / pressure shafts should remain fully submerged during the sudden startup of the units to avoid entrapment of air.

Drawdown, $y = v * \sqrt{\frac{l}{g} * \frac{A_p}{A_f}}$

where, v = velocity in penstock
A_p = Area of "
A_f = surface area of forebay
and l = Length of penstock.

Minimum submergence,

$$s = a v \sqrt{d} \quad m.$$

where, $a = 0.545$ for symmetrical flow
$= 0.725$ for asymmetrical flow.

Vortex free conditions:

Relative submergence, $\frac{s}{d} > 0.5 + 0.4 F_r$

where, F_r = Froud number $= \frac{v}{\sqrt{gd}} < 0.5.$

※ Design a forebay for supplying 25 m^3/s discharge to turbine through a 200 m long penstock of 3 m diameter.

soln:

Assuming time taken by the flow for full acceleration $t = 4$ minutes and velocity of flow in forbay $v = 0.2$ m/s with depth $h = 3$ m

width, $b = \frac{Q}{hv} = \frac{25}{3*0.2} = 41.67$ m

length, $l = \frac{V}{bh}$

where, V = storage volume $= Q*t = 25*4*60 = 6000\ m^3$

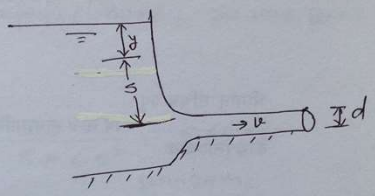
$\therefore l = \frac{6000}{41.67*3} = 48$ m. //

Now,

Draw down, $y = v\sqrt{\frac{1}{g}\frac{A_p}{A_f}}$

where, $v = \frac{Q}{A_p} = \frac{25}{\pi*\frac{3^2}{4}}$

$= 3.54$ m/s



$A_p = \pi\frac{d^2}{4} = \pi*\frac{3^2}{4} = 7.07\ m^2$

$A_f = l\,b = 48*41.67\ m^2$

$\therefore y = 3.54\sqrt{\frac{200}{9.81}*\frac{7.07}{48*41.67}} = 0.95$ m //

Minimum submergence,

$s = a\,v\sqrt{d}$ [Let the flow is symmetrical]

$= 0.545*3.54*\sqrt{3} = 3.34$ m. //

Vortex free conditions,

$\frac{s}{d} > (0.5 + 0.4\,Fr)$

$Fr = \frac{v}{\sqrt{gd}} = \frac{3.54}{\sqrt{9.81*3}} = 0.65 \not< 0.5$,

which is not so trouble in operation.

i.e., Required s/d = 0.5 + 0.4 × 0.65 = 0.76

Calculated s/d = 3.34/3.0 = 1.113 > Required s/d (O.K.)

Since y < h, additional depth is not required.

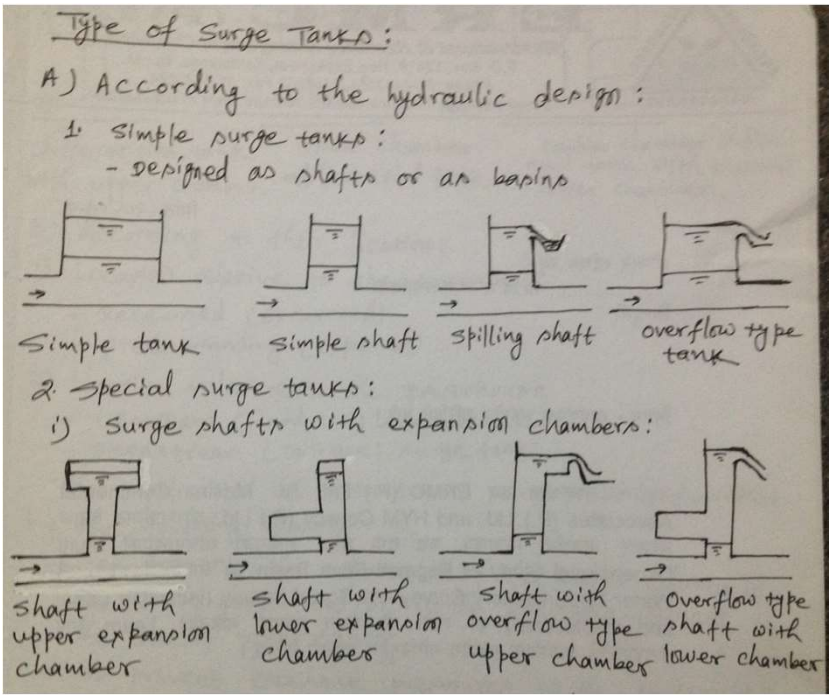
Surge tank or Expansion Chamber or Surge Chamber:

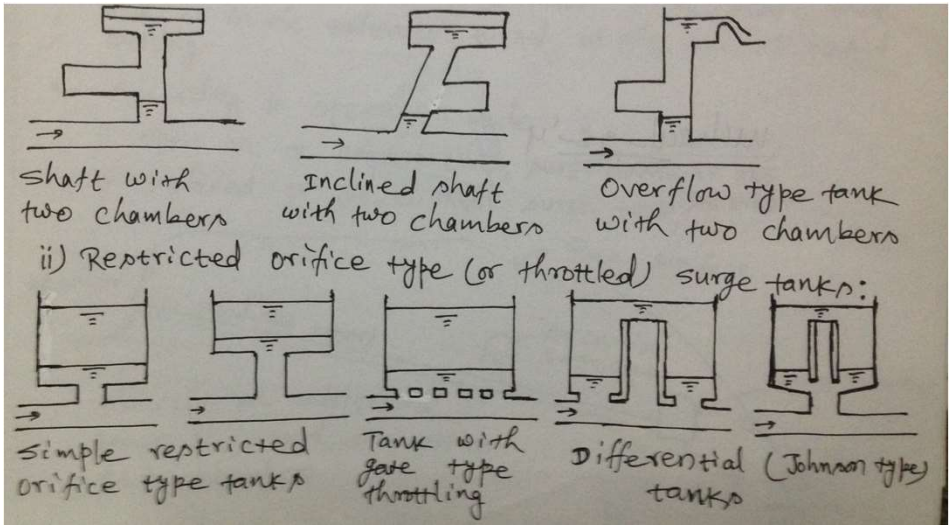
Surge tank is a hydraulic structure located between slightly inclined pressure conduit (tunnel) and steeply sloping penstock (pressure shaft) constructed to control hydraulic transient in a hydropower system.

It is designed as a chamber excavated in the mountain in most of the cases or as a tower raising high above the surrounding terrain.

Purpose of Surge Tanks:

- Intercepts or at least redically reduces the pressure surges due to water hammer and exempts thereby the pressure tunnel from excessive internal pressures.
- Provides protection to the penstock itself against the detrimental effects of water hammer if no bypass valve is installed or if bypass valve fails to operate.
- Improves the regulation of the governing system
- Provides additional water supply to the turbines in the case of starting up.


Type of Surge Tanks:
A) According to the hydraulic design:
1. Simple surge tanks:
- designed as shafts or as basins
Simple tank
Simple shaft
Spilling shaft
Overflow type tank
2. Special surge tanks:
i) Surge shafts with expansion chambers:
Shaft with upper expansion chamber
Shaft with lower expansion chamber
Shaft with overflow type upper chamber
Overflow type shaft with lower chamber

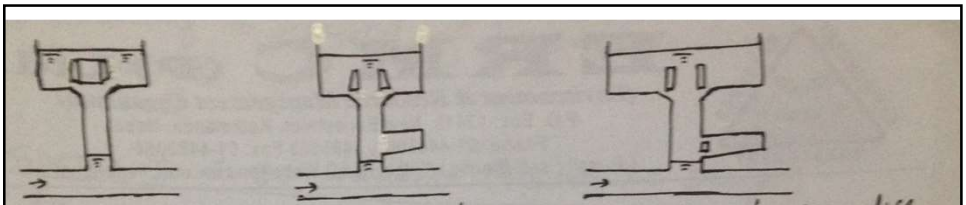

Shaft with two chambers
Inclined shaft with two chambers
Overflow type tank with two chambers
ii) Restricted orifice type (or throttled) surge tanks:
Simple restricted orifice type tanks
Tank with gate type throttling
Differential (Johnson type) tanks

Simple surge tanks: The simple surge tank is of uniform cross-section and is open to the atmosphere, acting as a reservoir. It is directly connected to the penstock so that water flows in and out with small head losses when load variations occur. It is usually large in size with expensive proportions and sluggish in responding to damping surges. These are very rarely used in modern practice except in installations where load changes are either small or very gradual.

Throttled tank: In the throttled tank (restricted orifice type tank) the restricted entry to the surge tank creates retardation and acceleration conditions of flow in the tunnel upstream of it, thus reducing the storage requirement and minimizing the maximum up and down surges. Although this type of surge tank is economical (because of its smaller size) compared with the simple tank section, the rapid creation of retarding and accelerating heads complicates the governing mechanism, requiring additional inertia in the turbo-generator units.

Surge tank with expansion chambers: This type of surge tank consists of a narrow riser (main surge shaft); attached to it at either end are large expansion chambers. The narrow riser reacts quickly, creating accelerating or decelerating heads, and at the same time the expansion chambers minimize the maximum up- and down-surge levels, thus limiting the range of surge levels (i.e. easier governing). In order to reduce the costs of the structure, spilling arrangements may sometimes be provided either to wastage (if water is not scarce) or back to the penstock.

Differential surge tank: This type (also known as Johnson's differential tank) consists of an internal narrow riser shaft with an orifice entry to the larger outer shaft at the bottom. As the central riser is narrow it responds instantaneously during the upward phase; at the same time the maximum amplitude is restricted to its top level, any excess water spilling back into the outer chamber. Similarly, during the downward phase water spills into the narrow riser while the riser itself responds quickly to maintaining the desired level. The differential tank with an extended penstock, which acts as a central riser.



Differential tank with upper chamber | Double chamber differential tank | Double chamber differential tank with throttled lower chamber

B) According to their location:

i) Location relative to the terrain
- Recessed (Excavated)
- Free-standing (Exposed)

ii) Location relative to the powerhouse
- Upstream (Headrace) surge tank
- Downstream (Tailrace) surge tank

The application of downstream surge tank is justified by the following reasons:

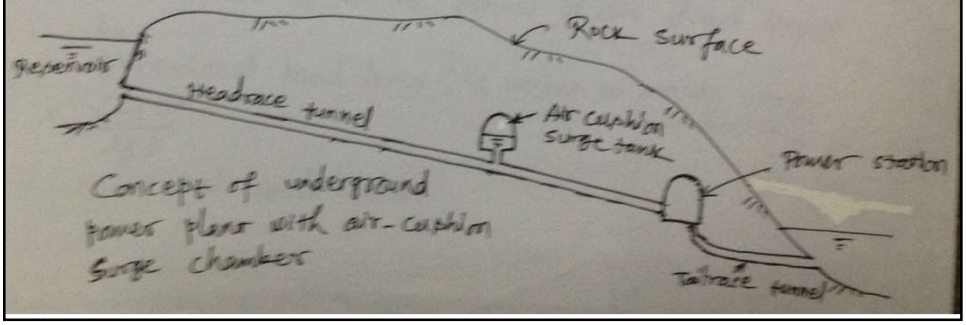
→ To prevent the separation of the water column in the draft tube and/or in the tailwater tunnel, in the case of rapid closure.

→ To prevent excessive pressures in the draft tube and or in the tailwater tunnel, in the case of rapid opening

C) According to operation mode:

i) Open air or atmospheric surge tank

ii) Enclosed or air-cushion surge tank



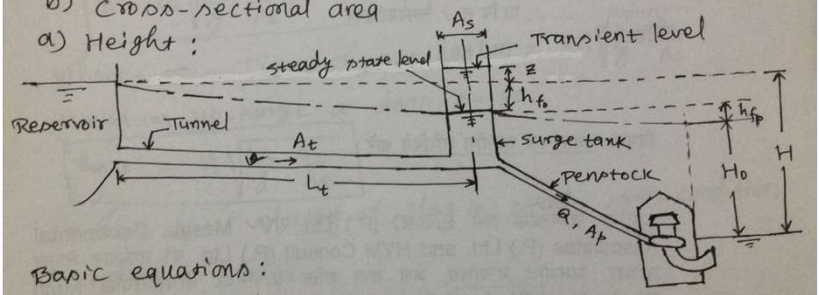
Concept of underground power plant with air-cushion surge chamber

D) According to material of construction:
- Concrete
- steel

Surges in surge chambers (Hydraulic design):

The hydraulic design of surge tank concerns itself with two main aspects
a) Height
b) Cross-sectional area

a) Height:



Basic equations:

$v A_t = A_s \frac{dz}{dt} + Q$ → Continuity equation

$\frac{L_t}{g}\frac{dv}{dt} + z \pm h_f = 0$ → Momentum equation

Where,

Q = Rate of flow to the power house at any time t

$\frac{dz}{dt}$ = Rate at which water rises in the surge tank

A_s = Cross-sectional area of surge tank

A_t = Cross-sectional area of headrace tunnel

v = Flow velocity in the tunnel of dia. d and length L_t.

h_f = Frictional head loss [h_{f_0} refers to steady state condition]

$= \frac{f L_t v^2}{2gd}$

f = Frictional coefficient

±ve sign = +ve sign for the flow direction from the reservoir and -ve sign for the flow direction to the reservoir.

The solution of the above equation gives the value of z at any time t.

i) For the simplest case of full closure and negligible friction,

$$z = v_0 \sqrt{\frac{L_t}{g} \frac{A_t}{A_s}} \sin 2\pi \frac{t}{T}$$

where, T = Time of oscillation = $2\pi \sqrt{\frac{L_t}{g} \frac{A_s}{A_t}}$

Maximum up-surge or down-surge,

$$z_{max} = v_0 \sqrt{\frac{L_t}{g} \frac{A_t}{A_s}}$$

where, v_0 = velocity of flow in steady state condition.

According to Novak,

$$z_{max} = \frac{Q_0}{A_s \cdot r} \quad \text{with } r = \sqrt{\frac{g A_t}{L_t A_s}}$$

ii) In cases where friction is taken into account,

$$\frac{z}{z_{max}} = 1 - \frac{2}{3} K_0 + \frac{1}{9} K_0^2$$, According to Jaegar for $K_0 < 0.7$

where, z = Maximum upsurge with friction taken into account

z_{max} = Maximum upsurge with negligible friction

$$K_0 = \frac{h_{f_0}}{z_{max}}$$

h_{f_0} = frictional head loss in the steady state condition.

$$\frac{z}{z_{max}} = -1 + 2K_0$$, According to Calame and Gaden for $K_0 < 0.8$

[Max^m downsurge when 100% opening i.e, 100% load demand]

where, z = Maximum downsurge with friction taken into account

z_{max} = Maximum downsurge with negligible friction

$$k_0 = \frac{h_{f_0}}{z_{max}}$$

* Upsurge to be determined considering maximum reservoir level and downsurge to be determined considering minimum reservoir level for the design of surge tank.

b) Cross-sectional area:

For the stability consideration, D. Thoma analysed assuming the following

- The turbine governer maintains a constant power output
- The surgetank oscillations are small
- The turbine efficiency is constant
- The friction losses correspond to steady state condition, and
- The governer response is instantaneous

He linearized the basic differential equation by neglecting the higher order non-linear terms, For damped oscillations

$$A_{sc} \geq \frac{v_0^2}{2g} \frac{A_t}{h_{f_0}} \frac{L_t}{H_0}$$

where, A_{sc} = critical cross-sectional area.

For actual practice, $A_s = FOS * A_{sc} = 1.5\ A_{sc}$.

For smooth concrete finishing,

$\frac{1}{n} = 85$ i.e, $n \approx 0.012$

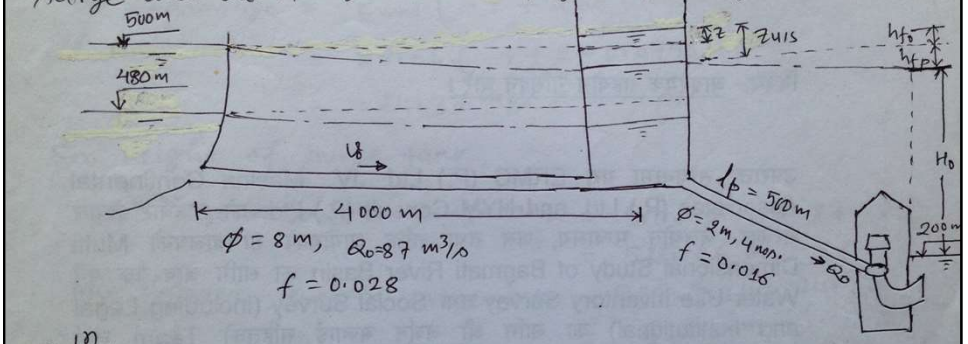
$$h_{f_0} = \frac{n^2 v_0^2 L_t}{R^{4/3}} \rightarrow \text{Manning's equation.}$$

Also, $R = d/4$, where d = dia. of tunnel

$$\therefore A_{sc} \geq \frac{v_0^2}{2g} \cdot \frac{\pi d^2}{4} \cdot \frac{(d/4)^{4/3}}{n^2 v_0^2 L_t} \cdot \frac{L_t}{H_0}$$

i.e., $$\boxed{A_{sc} \geq 45 \frac{d^{10}}{H_0}}$$

※ Determine cross-sectional area, maximum up-surge and downsurge and Height of surge tank.



Soln:

Frictional head loss from intake to surge tank

$$h_{f_0} = \frac{f l v^2}{2gD} = \frac{0.028 * 4000}{2 * 9.81 * 8} * \left(\frac{87}{\pi * \frac{8^2}{4}}\right)^2 = 2.14 \text{ m}$$

Frictional head loss from surge tank to power house

$$h_{fp} = \frac{flv^2}{2gD} = \frac{0.016 * 500}{2 * 9.81 * 2} * \left(\frac{87/4}{\pi * 2^2/4}\right)^2 = 9.78 \text{ m}$$

$$\therefore H_0 = 500 - 200 - h_{f_0} - h_{fp} = 300 - 2.14 - 9.78 = 288.08 \text{ m}$$

Now, $$A_{sc} = \frac{v_0^2 L_t A_t}{2g h_{f_0} H_0} = \left(\frac{87}{\pi * \frac{8^2}{4}}\right)^2 * \frac{4000 * \left(\pi \frac{8^2}{4}\right)}{2 * 9.81 * 2.14 * 288.08}$$

$$= 50 \text{ m}^2$$

$$\therefore A_s = 1.5 A_{sc} = 1.5 \times 50 = 75 \text{ m}^2 //.$$

Also,

$$Z_{max} = v_0 \sqrt{\frac{L_t}{g} \frac{A_t}{A_s}} = \left(\frac{87}{\pi * \frac{8^2}{4}}\right) \sqrt{\frac{4000}{9.81} * \frac{\pi * 8^2/4}{75}}$$

$$= 28.61 \text{ m}$$

$$K_0 = \frac{h_{f_0}}{Z_{max}} = \frac{2.14}{28.61} = 0.0748$$

$$\therefore Z_{upsurge} = Z_{max} \left[1 - \frac{2}{3} K_0 + \frac{1}{9} K_0^2\right]$$

$$= 28.61 \left[1 - \frac{2}{3} * 0.0748 + \frac{1}{9} * 0.0748^2\right]$$

$$= 27.20 \text{ m} //$$

and $$Z_{downsurge} = Z_{max} [-1 + 2K_0]$$

$$= 28.61 [-1 + 2 * 0.0748)$$

$$= -24.33 \text{ m} //$$

For height of surge tank,

Max^m water level in surge tank = 500 − 2.14 + 27.20
= 525.06 m

Min^m water level in surge tank = 480 − 2.14 − 24.33
= 453.53 m

Assuming FB = 2 m and water cushion at the bottom = 3 m

$\therefore$ Ht. of surge of tank = (525.06 − 453.53) + 2 + 3
= 76.53 m //

discharge

Penstocks:

Penstocks are the conveyance conduits which supply water from forebay / surgetank to turbines. These are high pressure conduits, designed to withstand water hammer pressure due to the change in boundary conditions such as starting and closing of turbine, sudden load rejection / acceptance, etc.

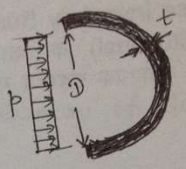
Classification of penstocks:

1. Based on material of fabrication
   - i) Steel
   - ii) Cast iron
   - iii) Plastic (for very low head)
   - iv) Wooden
   - v) RCC

2. Based on method of support:
   - i) Buried or embeded → Trench size 1 to 1.5 m deep
   - ii) Exposed above the ground
   - iii) Partly buried.
3. Based on rigidity of connections and support:
   - i) Rigid → Anchorage throughout the length i.e., no longitudinal movement
   - ii) Semi-rigid → Each member anchoraged (fixed) at one end only
   - iii) Flexible → Expansion joint throughout the length.

Thickness of penstock:

Acting force per unit length of penstock $= p * D * 1$ --- ①



Resisting force per unit length of penstock $= \sigma_{st} * 2(t * 1) * \eta_j$ ---- ②

where, D = Internal diameter of penstock
p = Internal pressure
σ_{st} = Permissible tensile stress in steel
η_j = Efficiency of welding at joints.

Equating equations ① and ②

$$p * D = \sigma_{st} * 2t * \eta_j$$

i.e; $$\boxed{t = \frac{p D}{2 \sigma_{st} \eta_j}}$$

or, $$\boxed{t = \frac{p R}{\sigma_{st} \eta_j}}$$

where, R = Internal radius of penstock

Add 1 to 3 mm generally for corrosion allowance.

ASME [American society of Mechanical Engineer] Recommendation:

$$\boxed{t = \frac{p R}{\sigma_{st} \eta_j - 0.6 p} + 0.15 \text{ cm}}$$

where, p = Internal pressure in kg/cm^2
R = Internal radius in cm
σ_{st} = Permissible tensile stress in steel in kg/cm^2
η_j = Efficiency of joint (0.9 to 1.0)

Diameter of penstocks:

1. Empirical formulae:

a) USBR (United States Bureau of Reclamation) formula:

$$v = 0.125\sqrt{2gH}$$

where, v = optimum velocity in m/s
and H = maximum working head in m

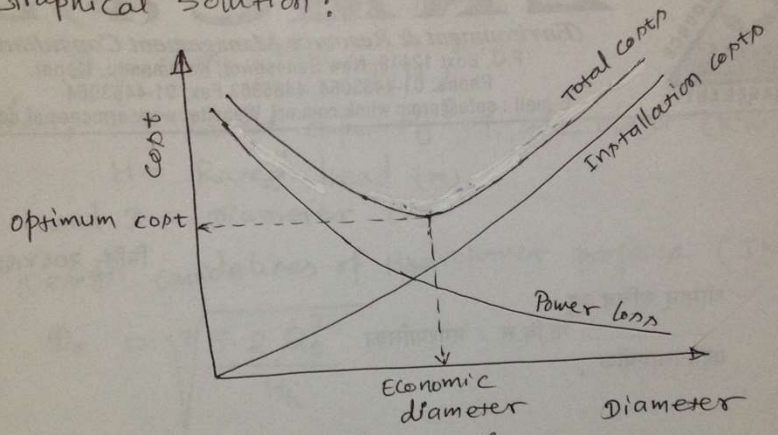
Then from continuity equation adopt diameter of penstock.

b) G. Sarkaria formula:

$$D = 0.62\frac{P^{0.35}}{H^{0.65}}$$

where, D = Penstock diameter in m
P = HP transmitted by the pipe
H = Maximum net head at the end of the penstock in m.

2. Graphical solution:



Economic diameter of Penstock

The energy of the flow through a penstock is inevitably reduced owing to entry and friction losses.

Although the friction losses can be minimized by careful selection of the pipe diameter, and its entrance losses can be minimized by a bell-mouthed entrance, an economical penstock diameter may be determined from a study of the annual charges on the cost of the installed pipe compared with the cost of revenue due to this power loss. Energy losses decrease with the increasing diameters while construction cost increase. A diameter which minimizes the total annual costs can be determined from the sum of the two costs.

Fahlbush (1982) reformulated the objective of the economic analysis in terms of the amount of the invested capital and the capitalized value of the lost energy, and arrived at the conclusion that the most economical diameter can be computed within an accuracy of about ± 10% from

$$D = 0.52\, H^{-0.17} (P/H)^{0.43}$$

where, P = Rated capacity of the plant (KW)
H = Rated head (m)
and D = Diameter (m)

3. Design Guidelines of Hydropower projects (JNN):

$$D_e = \sqrt[7]{\frac{5.2\, Q_t^3}{H_h}}$$

where, D_e = Economic diameter of penstock (m)
H_h = Dynamic head on the penstock (m)
Q_t = Flow discharge of the turbine (m^3/s)

4. Based on economic analysis:

$$d \leq \sqrt[7]{\frac{\lambda\, \sigma_a\, t\, C_2\, Q^3}{1000\, C_1\, H}}$$

where, d = economic diameter of penstock in cm
λ = friction coefficient
σ_a = allowable stress in kg/cm^2
t = annual duration of operation of HEP in hrs.
C_2 = unit cost of power generation
C_1 = annual cost of 1 kg wt. of penstock
H = design head in m
and Q = Discharge through penstock in m^3/s

* A penstock of discharging capacity of 5 m^3/s is to be functioned in a hydropower station with a design head of 50 m. Determine the economic diameter of the penstock. Take overall efficiency of the plant as 85%.

Soln:

$Q = 5$ m^3/s, $H = 50$ m, $\eta = 85\%$

$P = \gamma Q H \eta = 9.81 * 5 * 50 * 0.85 = 2084625$ watt

$= \frac{2084625}{746}$ HP $= 2794.4$ HP

Using USBR formula:

$v = 0.125\sqrt{2gH} = 0.125\sqrt{2 * 9.81 * 50} = 3.915$ m/s

$A = \frac{Q}{v}$

$\therefore D = \sqrt{\frac{4A}{\pi}} = \ldots = 1.28$ m//

Using Sorkaria formula

$D = 0.62\,\frac{P^{0.35}}{H^{0.65}} = 0.62 * \frac{(2794.4)^{0.35}}{50^{0.65}} = 0.784$m

Using formula based on Graphical solution

$$D = 0.52\, H^{-0.17} (P/H)^{0.43}$$

$$= 0.52 * 50^{-0.17} * \left(\frac{2084.625}{50}\right)^{0.43} = 1.51 \text{ m} //$$

* A penstock carries 8 m^3/s of water ~~at~~ head of 25 m. The cost of pipe line in place is given by $35\,hd^2$ rupees per metre length, where h is the head and d is the diameter of the pipe. Annual fixed charges are 8% of the pipe line cost. The head lost in friction is $0.025 Q^2/12.1 d^5$ m per m length of the pipe. Efficiency of the turbine is 80% and the selling price of the power generated is 70 rupees per kW per annum. Calculate the most economical diameter of the penstock.

Soln:

Consider 1 m length of the penstock.

Cost of pipe = Rs. $35 * 25 d^2$ = Rs. $875 d^2$

Annual fixed charges per annum

$= 0.08 * 875 d^2$ = Rs. $70 d^2$

Head lost = $\frac{0.025 Q^2}{12.1 d^5} = \frac{0.025 * 8^2}{12.1 * d^5} = \frac{0.132}{d^5}$

∴ Power lost = $9.81 * 8 * \frac{0.132}{d^5} * 0.80$ [$? QH\eta$]

$= \frac{8.29}{d^5}$

∴ Cost of power lost = Rs. $\frac{8.29}{d^5} * 70$ = Rs. $\frac{580.30}{d^5}$

∴ Total cost = Rs. $\left[70 d^2 + \frac{580.30}{d^5}\right]$

Differentiating this total cost with respect to d and equating to zero.

$$140 d - \frac{2901.5}{d^6} = 0$$

or, $140 d^7 = 2901.5$

∴ $d = 1.54$ m //.

Water hammer phenomena (Hydraulic Transient)

What is water hammer?

Water hammer is a phenomena that occurs in pressurized pipe systems when the flowing liquid is suddenly (instantaneously) obstructed by (for example) closing a valve or by pump stoppage (failure). A laud noise similar to hammer knocking noise occurs due to the collision of the liquid mass with the obstruction body (valve or pump) and the internal walls of the pipe.

The magnitude of pressure rise depends on:

(*i*) *The speed at which valve is closed,*
(*ii*) *The velocity of flow,*
(*iii*) *The length of pipe, and*
(*iv*) *The elastic properties of the pipe material as well as that of the flowing fluid.*

The rise in pressure in some cases may be so large that the pipe may even burst and therefore it is essential to take into account this pressure rise in the design of the pipes.

What is the problem with water hammer?

Hydraulic transient (water hammer) may cause disasters in pressurized liquid systems such as:

1. Rupture of pipes and pump casings.
2. Vibration and high noise.
3. Excessive pipe displacements.
4. Vapor cavity formation.
5. Environmental pollution.
6. Life and economical losses.

When water hammer occurs?

The most common cases where water hammer occurs are:

1. Turbine/pump failure.
2. Turbine/pump start up.
3. Sudden closure a valve.
4. Failure of flow or pressure regulators.

12·12·4. Time required by Pressure Wave to travel from the Valve to the Tank and from Tank to Valve.

Time taken, $$t = \frac{\text{Distance travelled from valve to tank and back}}{\text{Velocity of pressure wave}}$$

$$= \frac{L + L}{C} = \frac{2L}{C} \quad i.e., \quad t = \frac{2L}{C} \qquad ...(12·32)$$

where, L = Length of the pipe, and

C = Velocity of pressure wave.

(*i*) The closure of valve is said to be *gradual* when $t > \frac{2L}{C}$

(*ii*) The closure of valve is said to be *instantaneous* when $t < \frac{2L}{C}$

where, C = velocity of the pressure wave.

12·12·1. Gradual Closure of Valve

Consider a long pipe carrying liquid (Fig. (12·45)) and provided with a valve which is closed gradually.

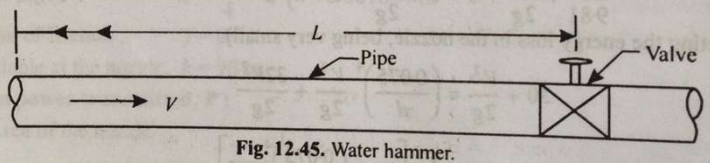


Fig. 12.45. Water hammer.

Let, A = Area of cross-section of the pipe,

L = Length of the pipe,

V = Velocity of flow of water in the pipe,

t = Time required to close the valve (in seconds), and

p = Intensity of pressure wave produced.

The mass of liquid contained in the pipe is $= \rho AL$

Assuming that the rate of closure of the valve is so adjusted that the liquid column in the pipe is brought to rest with a uniform retardation; from an intial velocity V to zero in time t seconds, we have,

Retardation of water $= \frac{V - 0}{t} = \frac{V}{t}$

$\therefore$ The axial force available for producing retardation

= Mass × retardation

$$= \rho AL \times \frac{V}{t} \qquad ...(i)$$

Also force due to pressure wave is $= p.A$ ...(*ii*)

Equating the two forces given by eqns. (*i*) and (*ii*), we have

$$\rho AL \times \frac{V}{t} = p \times A$$

or, $$p = \frac{\rho LV}{t} \qquad ...(12·26)$$

$\therefore$ Head of pressure, $$H = \frac{p}{w} = \frac{\rho LV}{w \times t} = \frac{\rho LV}{\rho \cdot g \cdot t} = \frac{LV}{gt}$$

i.e., $$H = \frac{LV}{gt} \qquad ...(12·27)$$

Instantaneous Closure of Valve in Rigid Pipes

Consider a pipe of length L and area of cross-section A (Fig. 12·45) carrying water which is flowing through it at a velocity V. When the valve is closed instantaneously the K.E. of the flowing water is converted into strain energy of water (neglecting effect of friction and assuming the pipe wall to be perfectly rigid).

Loss of $K.E. = \frac{1}{2} mV^2 = \frac{1}{2} \rho AL \times V^2$ $(\because m = \rho \times A \times L)$

Gain of strain energy $= \frac{1}{2}\left(\frac{p^2}{K}\right) \times \text{volume} = \frac{1}{2}\frac{p^2}{K} \times AL$

[where, k = Bulk modulus of elasticity of water, and
p = Intensity of perssure wave produced.]

Equating the loss of $K.E.$ to the gain of strain energy, we get

$$\frac{1}{2}\rho AL \times V^2 = \frac{1}{2}\frac{p^2}{K} \times AL$$

or, $$p^2 = \frac{1}{2}\rho ALV^2 \times \frac{2K}{AL} = \rho KV^2$$

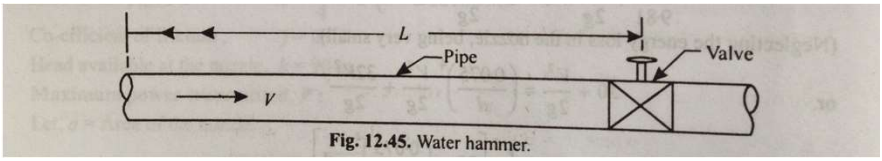


Fig. 12.45. Water hammer.

$$\therefore \quad p = \sqrt{\rho KV^2} = V\sqrt{\rho K} = V\sqrt{\frac{K\rho^2}{\rho}}$$

or, $$p = V\rho C \qquad ...(12·30)$$

$\left(\text{where, } C = \sqrt{\frac{K}{\rho}}, C \text{ being the } \textit{velocity of pressure wave.}\right)$

12·12·3. Instantaneous Closure of Valve in Elastic Pipes

As shown in Fig. 12·45, consider a pipe of length L, diameter D, thickness t (small compared to diameter).

Let, p = Increase of pressure due to water hammer,

E = Modulus of elasticity of pipe material, and

$\frac{1}{m}$ = Poisson's ratio for pipe material.

When the valve is closed intantaneously, rise of pressure takes place due to which circumferential and longitudinal stresses are produced in the pipe wall; these stresses are given as (from knowledge of strength of materials)

$$\sigma_c = \frac{pD}{2t} \text{ and } \sigma_l = \frac{pD}{4t}$$

where, σ_c = Circumferential stress, and

σ_l = Longitudinal stress.

Also, strain energy stored in the pipe material per unit volume is

$$= \frac{1}{2E}\left(\sigma_c^2 + \sigma_l^2 - \frac{2\sigma_c \sigma_l}{m}\right)$$

$$= \frac{1}{2E}\left[\left(\frac{pD}{2t}\right)^2 + \left(\frac{pD}{4t}\right)^2 - \frac{2 \times \frac{pD}{2t} \times \frac{pD}{4t}}{m}\right]$$

$$= \frac{1}{2E}\left[\frac{p^2D^2}{4t^2} + \frac{p^2D^2}{16t^2} - \frac{p^2D^2}{4mt^2}\right]$$

Assuming $\frac{1}{m} = 1/4$, we have

Strain energy per unit volume $= \frac{1}{2E}\left[\frac{p^2D^2}{4t^2} + \frac{p^2D^2}{16t^2} - \frac{p^2D^2}{16t^2}\right] = \frac{p^2D^2}{8Et^2}$

Total strain energy stored in pipe material

$$= \frac{p^2D^2}{8Et^2} \times \text{total volume of pipe material}$$

$$= \frac{p^2D^2}{8Et^2} \times \pi\, Dt \times L$$

$$= \frac{p^2 \times \pi D^2 \times DL}{8Et} = \frac{p^2 ADL}{2Et} \quad [\because A \text{ (area of the pipe)} = \frac{\pi}{4} \times D^2]$$

Loss of K.E. of water $= \frac{1}{2} mV^2 = \frac{1}{2} \rho AL \times V^2$

Gain of strain energy in water $= \frac{1}{2}\left(\frac{p^2}{K}\right) \times \text{volume} = \frac{1}{2}\frac{p^2}{K} \times AL$

Also, the loss of K.E. of water = gain of strain energy in water + strain energy stored in material.

$$\therefore \quad \frac{1}{2} \rho AL \times V^2 = \frac{1}{2}\frac{p^2}{K} \times AL + \frac{p^2 ADL}{2Et}$$

Dividing both sides by $\frac{AL}{2}$, we get

$$\rho V^2 = \frac{p^2}{K} + \frac{p^2 D}{Et} = p^2\left(\frac{1}{K} + \frac{D}{Et}\right)$$

$$\therefore \quad p^2 = \frac{\rho V^2}{\left(\frac{1}{K} + \frac{D}{Et}\right)}$$

or,
$$p = \sqrt{\frac{\rho V^2}{\left(\frac{1}{K} + \frac{D}{Et}\right)}} = V \times \sqrt{\frac{\rho}{\left(\frac{1}{K} + \frac{D}{Et}\right)}} \qquad ...(12{\cdot}31)$$

Example 12.51. *In a pipe 600 mm diameter and 3000 m length, provided with a valve at its end, water is flowing with a velocity of 2 m/s. Assuming velocity of pressure wave C = 1500 m/s, find :*

(*i*) *The rise in pressure if the valve is closed in 20 seconds, and*

(*ii*) *The rise in pressure if the valve is closed in 2·5 seconds. Assume the pipe to be rigid one and take bulk modulus of water as 2 GN/m².*

Solution. Diameter of the pipe, $D = 600$ mm $= 0{\cdot}6$ m

Length of the pipe, $L = 300$ m

Velocity of water, $V = 2$ m/s

Velocity of pressure wave, $C = 1500$ m/s.

(*i*) **Rise in pressure, p :**

Time taken to close the valve, $t = 20$ s

Now, the ratio, $$\frac{2L}{C} = \frac{2 \times 3000}{1500} = 4$$

The close of valve is said to be *gradual* if,

$$t > \frac{2L}{C}$$ ...[Eqn. (12·28)]

Hence, the valve is closed *gradually*.

The rise in pressure (p), for gradual closure of valve, is given by

$$p = \frac{\rho LV}{t}$$ ...[Eqn. (12·26)]

$$= \frac{1000 \times 3000 \times 2}{20} = 300 \times 10^3 \text{ N/m}^2 \text{ or } \mathbf{300 \text{ kN/m}^2 \text{ (Ans.)}}$$

(*ii*) **Rise in pressure, p :**

Time taken to close the valve, $t = 2{\cdot}5$ s

Bulk modulus of water, $K = 2$ GN/m²

Velocity of pressure wave is given by,

$$C = \sqrt{\frac{K}{\rho}} = \sqrt{\frac{2 \times 10^9}{1000}} = 1414{\cdot}2 \text{ m/s}$$

The ratio, $$\frac{2L}{C} = \frac{2 \times 3000}{1414{\cdot}2} = 4{\cdot}24 \text{ s}$$

$\therefore$ $$t < \frac{2L}{C}$$

Thus, the valve is closed *intantaneously* [From eqn. (12·29)]

When pipe is rigid, the rise in pressure due to instantaneous closure of the valve is given by (eqn. 12·30),

$$p = V\rho C = 2 \times 1000 \times 1414{\cdot}2 \text{ N/m}^2 \text{ or } \mathbf{2828{\cdot}4 \text{ kN/m}^2 \text{ (Ans.)}}$$

Example 12·52. *Water is flowing in a pipe of 150 mm diameter with a velocity of 2·5 m/s when it is suddenly brought to rest by closing the valve. Find the pressure rise assuming pipe is elastic, $E = 206$ GN/m², Poisson's ratio = 0·25 and K for water = 2·06 GN/m². Pipe wall is 5 mm thick.*

Solution. Diameter of the pipe, $D = 150$ mm $= 0{\cdot}15$ m

Thickness of the pipe, $t = 5$ mm $= 0{\cdot}005$ m

Velocity of water, $V = 2{\cdot}5$ m/s

Modulus of elasticity, $E = 206$ GN/m²

Bulk modulus of water, $K = 2{\cdot}06$ GN/m²

Poisson's ratio, $\frac{1}{m} = 1/4$

Pressure rise, p :

Using the relation :

$$p = V\sqrt{\frac{\rho}{\left(\frac{1}{K} + \frac{D}{Et}\right)}} \qquad \text{...[Eqn. (12·31)]}$$

$$= 2{\cdot}5\sqrt{\frac{1000}{\left(\frac{1}{2{\cdot}06 \times 10^9} + \frac{0{\cdot}15}{2{\cdot}06 \times 10^9 \times 0{\cdot}005}\right)}} = 2{\cdot}5\sqrt{\frac{10^{12}}{0{\cdot}485 + 0{\cdot}1456}}$$

= 3148 kN/m² (Ans.)

Example 12·53. *In a pressure penstock 4500 m long water is flowing at 4 m/s. If the velocity of the pressure wave travelling in the pipe due to sudden complete closure of a valve at the downstream end is given as 1500 m/s, find :*

(i) The maximum pressure rise, and

(ii) The period of oscillation.

Solution. Length of the penstock, $L = 4500$ m

Velocity of water, $V = 4$ m/s

Velocity of the pressure wave, $C = 1500$ m/s

(i) **The maximum pressure rise :**

Maximum pressure is given by

$$p = V\rho C \qquad \text{...[Eqn. (12·39)]}$$

$= 4 \times 1000 \times 1500$ N/m² or **6 MN/m² (Ans.)**

(ii) **The period of oscillation :**

The period of oscillation $= \frac{2L}{C} = \frac{2 \times 4500}{1500} =$ **6 seconds (Ans.)**

※ Determine the rise in pressure that will occur in a penstock leading to a power plant if the turbine gates are closed in 1.2 sec.

Initial head = 50 m

length = 1000 m

velocity = 3.6 m/s

Pressure wave velocity, c = 1429 m/s.

Soln:

we have, $T_c = \frac{2L}{c} = \frac{2*1000}{1429} = 1.4$ sec > 1.2 sec.

So it is the case of rapid ~~slower~~ closure.

i) In a rigid pipe:

$P_w = v\sqrt{\rho K} = v\sqrt{\rho^2 \frac{K}{\rho}} = v c \rho = 3.6 * 1429 * 1000$

$= 5144400$ kg/ms² or N/m²

$= 5144.40$ KN/m² //.

ii) In a elastic pipe:

$$\left[c = \sqrt{\frac{K}{\rho}} \right]$$

$$P_w = v\sqrt{\frac{\rho}{\frac{1}{K} + \frac{d}{Et}}}$$

※ A steel penstock 0.6 m in diameter has a shell thickness of 12 mm. The modulus of elasticity of the pipe shell material is $2.1 * 10^5$ N/mm², the volume modulus of elasticity is $2.1 * 10^3$ N/mm². The pipe is designed to discharge the water through the pipe of mean velocity of 2.1 m/s. Determine the water hammer pressure rise caused by sudden closure of the valve at the d/s a) neglecting the elasticity b) considering the elasticity.

Soln:

a) Neglecting the elasticity,

$P_w = v\sqrt{\rho K} = 2.1 * \sqrt{1000 * 2.1 * 10^9}$

$= 3.043 * 10^6$ N/m² //

$= 3.043 * 10^3$ KN/m² //.

b) Considering the elasticity.

$$p_w = v\sqrt{\frac{\rho}{\frac{1}{K}+\frac{d}{Et}}} = 2.1 * \sqrt{\frac{1000}{\frac{1}{2.1*10^9}+\frac{0.6}{2.1*10^{11}*12*10^{-3}}}}$$

$$= 2.485 * 10^6 \ N/m^2 \ //.$$

$$= 2.485 * 10^3 \ KN/m^2 \ //$$

※ What is the maximum permissible velocity in a cast iron pipe line 100 mm ϕ and 15 mm thick, which can be suddenly stopped by a valve at the outlet end of the pipe ~~whi~~ without letting the ripe in pressure exceed $1.545 * 10^3$ KN/m².

$E = 2.1 * 10^5 \ N/mm^2$

$K = 2.1 * 10^3 \ N/mm^2$

Soln:

$$p = v\sqrt{\frac{\rho}{\frac{1}{K}+\frac{d}{Et}}}$$

$$\therefore v = p\sqrt{\frac{\frac{1}{K}+\frac{d}{Et}}{\rho}}$$

$$= 1.545 * 10^6 \sqrt{\frac{\frac{1}{2.1*10^9}+\frac{0.10}{2.1*10^{11}*15*10^{-3}}}{1.050}}$$

$$= 1.101 \ m/s. \ //.$$

EXAMPLE 10.1

A penstock, with an internal diameter of 1.20 m, supplies water at a head equivalent to 17.6 kg/cm^2. There is a possibility of 20 per cent increase in the pressure due to transient conditions. The design stress and the efficiency of the joint may be assumed to be 1020 kg/cm^2 and 85 per cent respectively. Calculate the approximate wall thickness of the penstock required.

SOLUTION

Here total pressure intensity, including over pressure: $p = 17.6 + 3.52 = 21.12$ kg/cm^2

Design stress: $s = 1020$ kg/cm^2

Joint efficiency: $\eta = 85\%$

Internal radius: $R = 60$ cm

Wall thickness:

$$t = \frac{pR}{s\eta - (0.6\,p)} + 0.15 \text{ cm}$$

$$= \frac{21.12 \times 60}{1020 \times 0.85 - (0.6 \times 21.12)} + 0.15 \text{ cm}$$

$$= \frac{1267.2}{866 - 12.67} + 0.15 \text{ cm}$$

$$= 1.485 + 0.15 \text{ cm}$$

or $t = 1.635$ cm